APPENDIX 5-17

EVALUATION OF WAVE RUN-UP USING DESIGN CONDITIONS FROM DCM-2

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BLACK & VEATCH

TECHNICAL MEMORANDUM

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Evaluation of Wave Run-up Using Design Conditions from DCM-2

To: Shawn Waldeck and Rich Bartlett

From: Beth Quinlan

1. OBJECTIVE

The overall objectives of the Wave Run-up Model are as follows:

- To determine the amount of freeboard required to prevent over-topping of the reservoir embankment during high wind and rain conditions,
- To determine the effectiveness of internal breakwaters in decreasing wave run-up.

This memorandum summarizes additional modeling conducted to examine new design conditions and the effects of a perimeter bench. In addition, this memorandum addresses comments previously received on the wave run-up modeling. The Automated Coastal Engineering System (ACES) model was used to conduct this analysis. The USACE procedures relevant to this analysis include the following:

- Determine wave characteristics for each design case
- Determine the wind set-up
- Calculate the wave run-up
- Calculate overtopping rates for various embankment heights
- Calculate the effect that a perimeter bench has on wave run-up and overtopping

2. PREVIOUS WAVE RUN-UP MODELING

Results of previous wave run-up modeling were presented in Work Order 3, Technical Memorandum 4, Evaluation of Wave Run-up and Internal Breakwaters (Quinlan et al., 2005). That technical memorandum was submitted to the District on March 4, 2005 prior to the issuance of Design Criteria Memorandum: DCM-2 Wind and Precipitation Design Criteria for Freeboard. At the request of the District, the design conditions evaluated under Work Order 3 included:

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- 200 mph wind with no rainfall
- 105 mph wind with the Probable Maximum Precipitation (PMP)

Several other variables including fetch, depth, slope and type of surface on the embankment were modified to simulate a range of design conditions. The design conditions were evaluated with and without internal breakwaters that could reduce wave run-up on the embankments.

The wave growth and wave theory sections of the ACES model were used to identify the wave characteristics that could occur under the design conditions. Wind Set-up calculations were made for each of the cases evaluated using the Sibul Method. The wave-run-up module of the ACES model was used to estimate wave run-up for each of the cases evaluated.

Two configurations of internal breakwaters were evaluated; a peripheral wall approximately ½ mile inside of the embankment, and a circle breakwater in the middle of the reservoir with several spokes radiating toward the embankments. The results of the modeling indicated that the peripheral wall could reduce the embankment height by at least 4 to 7 feet. The circle breakwater would not be as effective at reducing freeboard and may reduce the embankment height by only about 1 foot.

3. DESIGN CONDITIONS

Wind and precipitation design conditions to be used on Acceler8 projects were developed and issued in draft form on March 21, 2005 (Haapala et al., 2005). Four design conditions were described in DCM-2 and were significantly different than those modeled under Work Order 3. Therefore, additional modeling was conducted to evaluate the new design conditions and to evaluate the effectiveness of a perimeter bench in reducing freeboard requirements. The design conditions that were modeled are described below. Additional details on developing the wind speeds and water levels to represent these design conditions, are presented in Appendix 5-17 Wave Run-up Case Descriptions.

3.1 Design Condition 1: 100 year Wind with PMP

The first design condition evaluated was a 100 year wind in combination with the Probable Maximum Precipitation (PMP) event. The 72 hour PMP for the reservoir was calculated to be 54 inches (Burgi et al., 2005). For this design condition, it was assumed that the PMP occurred first and then the wind event occurred. This is a conservative assumption because winds blowing over a deeper depth reservoir will produce higher waves.

DCM-2 presented information on 50-year to 100-year gust wind speeds and described methods for converting gust wind speeds to sustained winds. Wind gusts are not of sufficient duration to generate fully developed waves. Therefore, wind speeds should be adjusted from the time of observation (3 to 6 seconds for wind gusts) to the averaging time appropriate for wave generation. The averaging time will depend on the fetch of the reservoir.

The procedure described in DCM-2 was followed to determine the 100 year wind for the EAA A-1 Reservoir. According to figure DCM2-1 (Haapala et al., 2005), the 50-year, 3 second wind gust for the EAA A-1 Reservoir site is 125 mph. This number was converted to a 100-year 1-

hour wind speed of 107 mph. After adjustments for duration and overwater conditions, the final wind speed to represent this design condition was calculated to be 103 mph.

In summary, for Design Condition 1, the PMP was combined with a wind of 103 mph. If the PMP occurred at the normal maximum pool level, the water depth would be 16.5 ft. For this design condition, a wind speed of 103 mph was applied to the reservoir at a water depth of 16.5 ft.

3.2 Design Condition 2: 100 year Precipitation with Category 5 Hurricane

The second design condition includes rainfall that would occur during a 100 year storm in combination with a category 5 hurricane. According to DCM-2, a 1-minute wind speed of 156 mph should be used for this design condition. After adjustments for duration, the final wind speed to represent this design condition was calculated to be 122 mph. Using Figure DCM2-2 it was determined that the appropriate rainfall for this condition is 17 inches at the A-1 site. Therefore for Design Condition 2, a wind speed of 122 mph was applied to the reservoir at a water depth of 13.4 ft.

3.3 Design Condition 3: Probable Maximum Wind (200 mph)

The third design condition includes the probable maximum wind, that according to DCM-2, was 200 mph. This 200 mph wind speed was assumed to be an over-water, 1-minute average wind speed. The 1-minute wind speed was converted to a 1-hour average wind speed of 161 mph. Using Equation 4 of DCM-2 and considering the fetch of the A-1 reservoir, the 161 mph wind speed was converted to 158 mph and was applied to the reservoir at a water depth of 12 ft.

3.4 Design Condition 4: Storm Specific Wind and Precipitation

The fourth design condition is based on recorded data from Hurricane Easy which occurred in Florida in 1950. A maximum 3-second gust wind speed of 125 mph was recorded during the hurricane. This wind speed was converted using the procedures outlined in DCM-2 to a final wind speed of 96 mph. During Hurricane Easy, a peak 24-hour rainfall total of 38.7 inches was recorded. For this design condition, a wind speed of 96 mph was applied to the reservoir at a water depth of 15.2 ft. Because the wind speed and water depth for design condition 4 are both less than those of design condition 1, the required freeboard for this design condition would be less than that required under design condition 1. Therefore, this condition was not modeled using the ACES program.

3.5 Other Variables

In addition to the three design conditions modeled, two embankment types were simulated as well as the effects of a perimeter bench. Characteristics for the zoned embankment included 3:1 (H:V) side slopes and a rough surface. Roughness coefficients for rip-rap were used in the modeling. Characteristics for the RCC embankment included a vertical wall with a smooth surface. In all cases it was assumed that the perimeter bench would have a 3:1 slope and a rough surface. Bench widths of 25 ft and 15 ft were simulated.

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Previous modeling examined the effects of variations in fetch distance and water depth on wave growth and run-up. Results of previous modeling are included in Work Order No. 3, Technical Memorandum 4, Evaluation of Wave Run-up and Internal Breakwaters (Quinlan et al., 2005). The normal maximum water level for the A-1 reservoir is expected to be about 12 ft or slightly deeper. In all cases presented in this technical memorandum, the longest effective fetch was used as well as a normal maximum water level of 12 ft.

4. MODEL CONFIGURATION

The ACES (Automated Coastal Engineering System) program was used to calculate wave growth, wave run-up, and wave transmission over the perimeter bench. The calculations were completed using the wave prediction, wave theory, and wave run-up modules of the program. The ACES model does not calculate wind set-up and this was calculated separately using the Sibul model. Additional information on the model configuration, model calibration, verification and reliability is provided in a previous document (Quinlan et al., 2005) and in the Wave Run-up Model Documentation Memorandum (Quinlan, 2005).

4.1 Wave Prediction

The wave prediction section of the model computes wave growth. Wave growth is a function of the speed and duration of winds, fetch distance, and water depth. The outputs produced by the model include the effective fetch, adjusted wind speed, mean wave direction, wave height, and wave period.

4.2 Wind Set-Up

Wind set-up can be an important factor in determining freeboard requirements. Wind set-up occurs when wind blows in a relatively constant direction over the water surface. Shear stresses between the wind and water exert a drag on the water and push the water in the direction of the wind. When the water encounters a barrier such as a shoreline or embankment it piles up resulting in deeper water at the shoreline. Because the mass of water in the reservoir will be conserved, a decrease in water depth will occur at the leeward side of the reservoir to offset the wind set-up. However, the slope of the water surface is curved, not linear, so the decrease in depth at the leeward side of the reservoir will not equal the increase in depth at the windward side of the reservoir.

Wind set-up will increase until there is a balance between the shear stresses on the water surface and a gravity induced return flow along the reservoir bottom. Wind set-up is a function of wind speed, fetch, and water depth. Wind set-up increases with wind speed and fetch but decreases with increasing water depth.

Wind set-up is not included in the ACES model. The Sibul model was used to calculate wind set-up (USACE, 2004) and the results were added to the wave run-up calculations. The Sibul model is an empirical relationship based on the numerical model developed by Brater et al. (1996). The empirical relationship used laboratory and field data including data collected at Lake Okeechobee. The design conditions evaluated in this Technical Memorandum were not represented in the data used to develop the empirical relationship. Therefore, there is uncertainty in using the model for very high wind speeds. The following excerpt from USACE (2004) discusses the reduction in drag coefficient observed by Powell et al. (2003).

> *Experiments with high wind speeds over saltwater show an unexpectedly* [sic] *drop in the drag coefficient as speeds increase from approximately 90 mph to 114 mph (Powell). A possible explanation as suggested was that as wind speeds increase above hurricane force, the surface becomes layered in foam that may impede the transfer of momentum from the wind, essentially creating a "slip" surface. The reduced wind drag coefficient as observed appears to decrease to a range from 2.5 x 10^{-6} to 2.0 x 10^{-6} with the lowest expectation around 1.5 x 10^{-6}. . . . Remembering that these experiments were on saltwater, it is uncertain at this time whether or not these same observations may be expected on freshwater reservoirs.*

Although the experiments were conducted on saltwater it is likely that the organic content of the A-1 reservoir would be sufficiently high to produce foam during hurricanes. The conditions observed by Powell were for winds up to 114 mph. The probable wind condition modeled in this project is 158 mph. Therefore, it seems likely that some reduction in drag would occur and the wind set-up would not be as high as estimated. However, as a conservative assumption, no reduction in drag was included in the calculation of freeboard requirements.

4.3 Wave Run-Up

The wave run-up section of the model calculates the run-up that occurs when waves encounter a shoreline or embankment. The required inputs include wave type, breaking criteria, wave height, wave period, structure slope, structure height, slope type, and roughness coefficient. This section of the model also calculates overtopping rates.

The output calculated by the model includes wave run-up, deepwater wave height, and wave steepness. Figure 1 indicates how the wave run-up parameters are defined. Wave run-up (R) is measured from the still water level as opposed to wave height (H), which is measured from trough to crest.

5. RESULTS

5.1 Wave Characteristics

The wave growth section of the ACES model was used to identify the wave characteristics that could occur under the design conditions. The wave characteristics of wave height and wave period are summarized in Table 1 for all cases without a perimeter bench. The effective depth is the sum of the normal maximum operating level and rainfall and was used to generate wave characteristics including wave height. The wave height and wave period increase with increasing fetch, depth and wind speed. Wave heights for the design conditions ranged from 6.6 to 7.1 ft.

5.2 Wind Set-up

Wind set-up calculations were made for each of the cases evaluated. Results of these calculations are presented in Table 2. Wind set-up increases with increasing wind speed and fetch. Wind set-up decreases with increasing depth. Wind set-up for the design conditions ranged from 2.1to 7.0 ft.

5.3 Wave Run-up

The wave-run-up module of the ACES model was used to estimate wave run-up for each of the design conditions evaluated. The results of the wave run-up modeling for the zoned and RCC embankments are presented in Tables 3 and 4. Each table lists the wind speed, wave height, rainfall amount, effective depth, wave run-up, wind set-up, and maximum water level. The wave run-up is not directly related to water depth and therefore is not related to wind set-up. Wave run-up is indirectly related to water depth because depth affects the incident wave height. The maximum water level is the sum of the effective depth, wind set-up and wave run-up.

5.4 Effects of Perimeter Bench

Modeling was also conducted to determine the effectiveness of a perimeter bench on reducing wave run-up, thereby reducing freeboard requirements. Wave run-up modeling conducted under Work Order 3 showed that an internal breakwater could be effective at reducing wave run-up by causing the waves to break prior to reaching the embankment. However, these structures would be very large and not cost-effective.

Modeling conducted by the USACE (Hadley, 2005) showed that a 25 ft wide perimeter bench would break waves and could significantly reduce the required freeboard. Modeling conducted for the C-43 reservoir showed that a bench submerged at a depth of 3 ft below the maximum surcharge depth would reduce wave heights to about one third of the incident wave height and reduce waver periods by about ten percent.

The ACES model can not simulate the effects of a submerged bench. With the wind set-up, water depth could be as high as 19 ft before adding wave run-up. For modeling purposes, the bench was set at a depth of 19.05 ft. It is recognized that a bench at a lower depth would likely be just as effective. Modeling to optimize the bench depth will be conducted during preliminary design.

For the purposes of this BODR it was assumed that the bench would be 19 ft above the reservoir bottom. This was simulated in the ACES program by first modeling the incident wave on an impermeable breakwater with a height of 19 ft, a width of 25 feet, and 3:1 slide slopes covered with rip-rap. The transmitted wave characteristics were then used as the wave characteristics that would run-up on the embankment. In summary, the steps taken to calculate the final wave run-up were:

1. Use the wave prediction module to calculate the characteristics of the incident wave.
2. Apply the incident wave to an impermeable breakwater with a height of 19 ft and calculate the characteristics of the transmitted wave.
3. Apply the transmitted wave to the zoned and RCC embankments.

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Results of the cases that include a 25 ft bench are presented in Table 5. The bench has a 3:1 slope, a width of 25 ft, and is 19 ft high, above original ground level. The transmitted wave height and period describe the characteristics of the wave that would be running up on the embankment.

It is possible that the need for rip-rap covering the embankment could reduce the width of the bench. Additional cases for a 15 ft wide bench for both the Zoned and RCC embankments were modeled and the results are presented in Table 6.

A perimeter bench would be very effective in reducing wave run-up on the embankment. For the two cases where the water depth is at the approximate height of the bench, the transmitted wave is about one third the height of the incident wave. This is approximately the same ratio calculated by the USACE in their modeling of the C-43 reservoir. A submerged bench would also be effective in breaking the incident wave and reducing wave run-up.

5.5 Overtopping Analysis

The ACES model was also used to calculate overtopping rates for the three design cases and both embankment types. Overtopping rates were calculated in one-foot increments starting at the Maximum Water Level and continuing until the overtopping rate was less than 0.0005 ft^3/sec/ft. Although a decision was not reached, at the CCM meeting on June 21, 2005 there was some discussion of using the rate of 0.01 ft^3/sec/ft as being equivalent to zero. The Maximum Water Level is the sum of the effective depth, wind set-up and wave run-up. At this level there would be no overtopping for a monochromatic wave field. An overtopping rate was not calculated (NC) for any case where the embankment height was less than the maximum water level.

The overtopping analysis was conducted assuming irregular waves. This recognizes that wind generated waves are not uniform and that a small percentage of waves will run-up higher onto the embankment than the predicted height. Table 7 presents the overtopping analysis for the embankments without a perimeter bench. Tables 8 and 9 present the overtopping analysis for embankments with a 25 ft wide and 15 ft wide perimeter bench, respectively.

6. SUMMARY AND CONCLUSIONS

Wind and precipitation design conditions to be used on Acceler8 projects were developed and issued in draft form on March 21, 2005 (Haapala et al., 2005). Four design conditions were described in DCM-2. Additional modeling was conducted to evaluate the new design conditions and to evaluate the effectiveness of a perimeter bench in reducing freeboard requirements.

The first design condition is defined as a 100 year wind in combination with the Probable Maximum Precipitation (PMP) event. For Design Condition 1 a wind speed of 103 mph was applied to the reservoir at a water depth of 16.5 ft. The second design condition includes rainfall that would occur during a 100 year storm in combination with a category 5 hurricane. For Design Condition 2, a wind speed of 122 mph was applied to the reservoir at a depth of 13.4 ft. The third design condition includes the probable maximum wind (200 mph) at the normal maximum reservoir depth. For Design Condition 3, a wind speed of 158 mph and was applied to the reservoir at a depth of 12 ft. The fourth design condition represented conditions that would produce a lower maximum water level than design condition 1 and was therefore, not modeled using the ACES program.

In addition to the design conditions discussed above, two embankment types were simulated as well as the effects of a perimeter bench. Characteristics for the zoned embankment included 3:1 (H:V) side slopes and a rough surface. Roughness coefficients for rip-rap were used in the modeling. Characteristics for the RCC embankment included a vertical wall with a smooth surface. In all cases it was assumed that the perimeter bench would have a 3:1 slope and a rough surface. Bench widths of 25 ft and 15 ft were simulated.

The wave growth section of the ACES model was used to identify the wave characteristics that could occur under the design conditions. Wave heights for the design conditions ranged from 6.6 to 7.1 ft. Wind set-up calculations were made for each of the cases evaluated. Wind set-up for the design conditions ranged from 2.1 to 7.0 ft. The wave-run-up module of the ACES model was used to estimate wave run-up for each of the design conditions evaluated. The maximum water level is the sum of the effective depth, wind set-up and wave run-up. The maximum water level ranges from 23.1 to 25.7 ft, for the zoned embankment and from 24.8 to 27.5 ft for the RCC embankment.

Modeling was also conducted to determine the effectiveness of a perimeter bench on wave run-up thereby reducing the required freeboard. The ACES model can not simulate the effects of a submerged bench. For modeling purposes, the bench was set at a depth of 19.05 ft. A perimeter bench would be very effective in reducing wave run-up on the embankment. For a 25 ft wide bench the maximum water level ranges from 21.1 to 22.0 ft, for the zoned embankment and from 20.8 to 21.9 ft for the RCC embankment. For a 15 ft wide bench the maximum water level ranges from 21.3 to 22.4 ft, for the zoned embankment and from 21.8 to 22.4 ft for the RCC embankment. A submerged bench would also be effective in breaking the incident wave and reducing wave run-up Modeling, to optimize the bench depth, will be conducted during preliminary design.

The ACES model was also used to calculate overtopping rates for the three design cases and both embankment types. Overtopping rates were calculated in one-foot increments starting at the Maximum Water Level and continuing until the overtopping rate was less than 0.0005 ft^3/sec/ft. An overtopping rate was not calculated (NC) for any case where the embankment height was less than the maximum water level. The overtopping analysis indicates that the embankment height can be significantly reduced with the addition of a perimeter bench.

7. REFERENCES

Brater, E. F., H.W. King, J.E. Lindell, and C.Y. Wei, 1996. “Handbook of Hydraulics,” Mc-Graw-Hill Company, Inc.

Burgi, K., S. Ramos, and J. Schlaman, 2005. “Task 5.3.4.4.3 Interim Summary Technical Memorandum, Evaluation of PMP/PMF and Hydrologic – Draft,” In; Basis of Design Report, EAA A-1 Reservoir, Appendix 5-2.

Haapala, John, Terry Arnold, and Yung Shen, 2005. "Design Criteria memorandum:DCM-2, Draft to CERP Team, March 21, 2005.

Hadley, Lori, June 2005. "C-43 Reservoir Internal Wave Break Optimization," USACE, Jacksonville district. Draft.

Leenknecht, David A., Andre Szuwalski, and Ann R. Sherlock, September 1992. "Automated Coastal Engineering System: Technical Reference," Coastal Engineering Research Center. Department of the Army. Vicksburg, MS.

Madsen, O. S. and S. M. White, 1976. "Reflection and Transmission Characteristics of Porous Rubble-Mound Breakwaters," CERC MR 76-5, US Army Engineer Waterways Experiment Station. Vicksburg, MS.

Powel, M.D., P.J. Vickery, and T.A. Reinhold, March 2003. "Reduced Drag Coefficient for High Wind Speeds in Tropical Cyclones." Nature, Vol. 422.

Quinlan, B., 2005. "Task 5.3.5.2.2 Model Documentation Memorandum For Wave Run-up Model – Draft" In Basis of Design Report, EAA A-1 Reservoir, Appendix 5-15.

Quinlan, B., J. Schlaman, and S. Ramos, 2005. "Evaluation of wave Run-up and Internal Breakwaters - Draft" In: Basis of Design Report, EAA A-1 Reservoir, Appendix 5-13.

Seelig, W. N., 1980. "Two-Dimensional Tests of Wave Transmission and Reflection Characteristics of Laboratory Breakwaters," 1980. CERC TR 80-1, US Army Engineer Waterways Experiment Station. Vicksburg, MS.

United States Army Corps of Engineers, 2004. "Analyses and Hydraulic Design of Embankment Heights for Aboveground Impoundments and Reservoirs for Projects of the Comprehensive Everglades Restoration Plan: Working Draft." Jacksonville District US Army Corps of Engineers. 4 December 2004.

Evaluation of Wave Run-up Using Design Conditions in DCM-2

TABLES

Table 1 Wave Characteristics

Wind Speed (mph)	Effective Depth (ft)	Fetch (miles)	Wave Height (ft)	Wave Period (sec)
103	16.5	8.3	6.65	5.17
122	13.4	8.3	6.53	5.36
158	12.0	8.5	7.06	5.81

Table 2 Wind Set-up Calculations

Wind Speed (mph)	Fetch (miles)	Effective Depth (ft)	Wind Set-up (ft)	Water Depth (ft)
103	8.3	16.5	2.1	18.6
122	8.3	13.4	3.6	17.0
158	8.3	12.0	7.0	19.0

Table 3 Wave Run-up Results, Zoned Embankment 3:1 Slope, Rough Surface

Wind Speed (mph)	Wave Height (ft)	Rainfall Depth (ft)	Effective Depth (ft)	Wave Run-up (ft)	Wind Setup (ft)	Maximum Water Level (ft)
103	6.65	4.5	16.5	6.0	2.1	24.6
122	6.53	1.4	13.4	6.1	3.6	23.1
158	7.06	0.0	12.0	6.7	7.0	25.7

Table 4 Wave Run-up Results, RCC Embankment Vertical Slope, Smooth Surface

Wind Speed (mph)	Wave Height (ft)	Rainfall Depth (ft)	Effective Depth (ft)	Wave Run-up (ft)	Wind Setup (ft)	Maximum Water Level (ft)
103	6.65	4.5	16.5	7.9	2.1	26.5
122	6.53	1.4	13.4	7.8	3.6	24.8
158	7.06	0.0	12.0	8.5	7.0	27.5

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Table 5 Results of Cases With a 25 ft Bench

Wind (mph)	Embankment Slope, Surface	Water Depth [a] (ft)	Transmitted Wave		Wave Run-up (ft)	Maximum Water Level [b] (ft)
			Height (ft)	Period (sec)		
103	3:1, rough	18.6	2.27	4.6	2.6	21.6
122	3:1, rough	17.0	1.60	4.8	2.1	21.1
158	3:1, rough	19.0	2.55	5.2	3.0	22.0
103	vertical, smooth	18.6	2.27	4.6	2.5	21.5
122	vertical, smooth	17.0	1.60	4.8	1.8	20.8
158	vertical, smooth	19.0	2.55	5.2	2.9	21.9

Notes:
a: Water Depth is the sum of the normal maximum level, rainfall, and the wind set-up
b: Wave Run-up heights were added to the 19 ft bench depth

Table 6 Results of Cases With a 15 ft Bench

Wind (mph)	Embankment Slope, Surface	Water Depth [a] (ft)	Transmitted Wave		Wave Run-up (ft)	Maximum Water Level [b] (ft)
			Height (ft)	Period (sec)		
103	3:1, rough	18.6	2.27	4.6	2.9	21.9
122	3:1, rough	17.0	1.60	4.8	2.3	21.3
158	3:1, rough	19.0	2.55	5.2	3.4	22.4
103	vertical, smooth	18.6	2.27	4.6	3.0	22.0
122	vertical, smooth	17.0	1.60	4.8	2.1	21.1
158	vertical, smooth	19.0	2.55	5.2	3.4	22.4

Notes:
a: Water Depth is the sum of the normal maximum level, rainfall, and the wind set-up
b: Wave Run-up heights were added to the 19 ft bench depth

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Table 7 Results of Overtopping Analysis Cases Without a Bench ($ft^3/sec/ft$)

Embankment Height (ft)	158 mph		122 mph		103 mph	
	Zoned	RCC	Zoned	RCC	Zoned	RCC
26	0.270	NC	0.008	0.104	0.001	NC
27	0.105	NC	0.002	0.041	0.000	0.146
28	0.037	0.027	0.000	0.015		0.060
29	0.015	0.012		0.001		0.024
30	0.003	0.005		0.000		0.008
31	0.000	0.002				0.002
32		0.000				0.000
NC - a value was not calculated because the embankment height was less than the Maximum Water Level						

Table 8 Results of Overtopping Analysis Cases with a 25 ft Wide Bench (ft3/sec/ft)

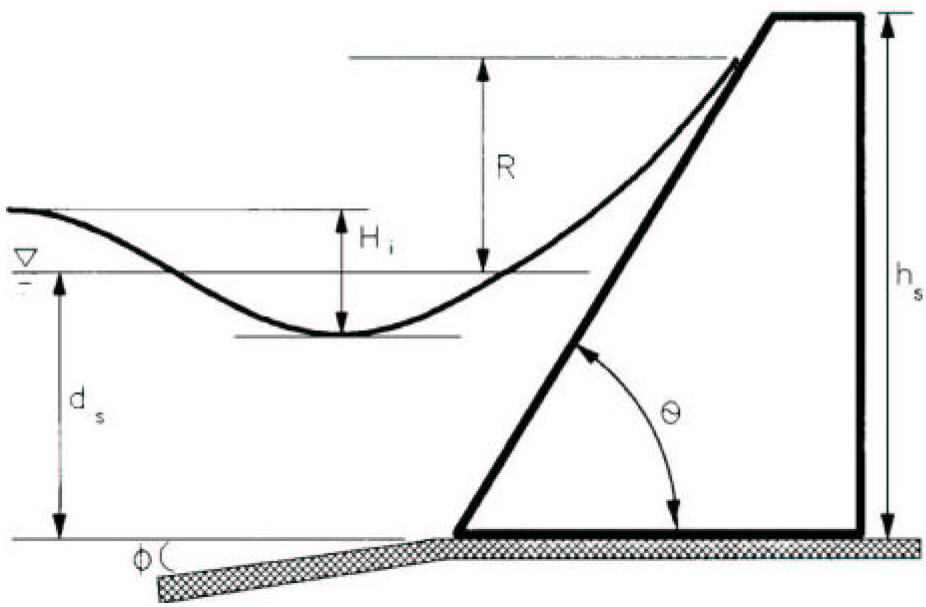
Embankment Height (ft)	158 mph		122 mph		103 mph	
	Zoned	RCC	Zoned	RCC	Zoned	RCC
22	0.086	0.081	0.001	0.000	0.017	0.013
23	0.010	0.006	0.000		0.001	0.001
24	0.000	0.000			0.000	0.000

Table 9 Results of Overtopping Analysis Cases with a 15 ft Wide Bench (ft3/sec/ft)

Embankment Height (ft)	158 mph		122 mph		103 mph	
	Zoned	RCC	Zoned	RCC	Zoned	RCC
22	NC	NC	0.005	0.001	0.037	0.043
23	0.036	0.039	0.000	0.000	0.004	0.004
24	0.004	0.004			0.000	0.000
25	0.000	0.000				
NC - a value was not calculated because the embankment height was less than the Maximum Water Level						

FIGURES

Figure 1 Definition of Wave Run-Up Parameters



(Leenknecht et al., 1992)